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Grouted connections on offshore wind turbines: a review

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Grouted connections (GCs) have been extensively used in offshore applications over the last decades and are widely used today in offshore monopile wind turbines. The effectiveness of the connections on monopiles was questioned recently after several substructures were reported to have insufficient performance in wind farms over Europe. This paper brings together the current practice in terms of engineering methods used for the determination of loads acting on the structure and the numerical methods used for the investigation of the structural behaviour of the GC. With respect to hydrodynamic loads on monopile wind turbines, the popular linear wave theory along with the Morison equation can be used to model normal sea states, whereas higher-order wave models are necessary to investigate severe events such as wave breaking. In terms of wind loads, blade element momentum proves to be advantageous with respect to computational cost and ease of implementation in simulation tools. Finally, finite-element modelling of GCs is introduced and close attention is given to the non-linearity of the grout material and the interface contact which are considered to be decisive aspects for the structural analysis.

1. Introduction

The need to confront the global challenges of fossil fuel depletion, climate change and increasing energy demand has led to the development of clean energy technologies that are used for power generation. In the UK, offshore wind energy has grown significantly as it is considered the main contributor for energy production from renewable energy sources (Higgins and Foley, 2014). An offshore wind turbine (OWT) can be mounted on top of different types of support structures. The vast majority of the substructures used in OWTs in Europe are fixed bottom structures known as monopiles (EWEA, 2014). Nowadays in the UK, there are 24 operating offshore wind farms at water depths of up to 30 m, where monopiles (Figure 1) are the most common foundation used (RenewableUK, 2015). It is formed from two cylindrical steel tubes, the transition piece (TP) and the monopile, which are attached with a grouted connection (GC). The TP is usually of larger diameter than the monopile, mainly to accommodate operations with the platform, such as boat landing. The tower and the turbine are installed on top of the substructure. GCs in the offshore industry and more specifically in the oil and gas sector have been widely used to support subsea structures (e.g. Gjersoe *et al.*, 2011; Löhning *et al.*, 2013). Lotsberg (2013), among others, stated that the performance of these steel–cementitious grout joints is known to be satisfactory for oil and gas applications and that is the reason why the GCs were used in the OWTs in order to connect the monopile with the TP.

Lotsberg *et al.* (2012) noted that the connection in OWTs is exposed to bending moments of significant magnitude, and the loads on the structure differed from the ones accounted for marine structures. Implications of several wind farms (e.g. Kentish flats and Horns Rev I), such as settlements and abrasive wear due to sliding between the surfaces in contact have been reported recently (e.g. Dallyn *et al.*, 2015; Schaumann *et al.*, 2010), indicating that the performance of the connection is inadequate. The reported insufficient capacity is closely related to uncertainties in the determination of loads acting on the structure and the design of the connection itself. The purpose of this review paper originates from the aforementioned implications and aims to address the state of the art on the experimental and numerical methods used to determine the loads acting on monopiles along with the methods used to investigate the behaviour of the connection.

2. Environmental loading

Understanding the loads acting on monopile OWTs is of significant importance in order to address the issues related

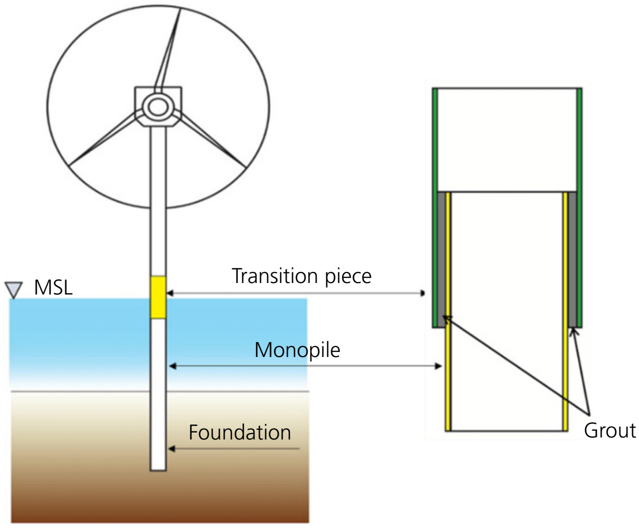


Figure 1. Monopile layout with GC (after DNV, 2014)

to GCs. The dominant loads acting on OWTs are those induced by waves and wind and are often referred to as environmental. The distinction of this section is based on the type of load acting on the structure. Therefore, hydrodynamics along with aerodynamics are addressed.

2.1 Wave theories

The simplest way to address ocean waves is to treat them as linear regular waves. In its broadest sense, a regular wave is a wave that has a period for which all cycles are of the same form. Regarding linear waves, potential flow theory is the basis. An irrotational and inviscid fluid flow along with incompressibility is assumed and the pressure is governed by the Bernoulli equation. All the aforementioned assumptions lead to the Laplace equation which is satisfied by the velocity potential (e.g. Newman, 1977). The solution of the Laplace equation with linear boundary conditions yields the analytical expressions for linear waves. It forms the fundamental principle for a deterministic representation of the sea state, but is limited by the fact that it is only valid for very small wave heights (e.g. $H/\lambda \ll 1$ and $d/\lambda < 0{\cdot}03$ where H is the wave height, λ the wavelength and d is the water depth, Dean and Dalrymple, 1992). For further details on the derivation of linear wave theory and its applicability see, for example, Faltinsen (1990), Newman (1977) and Sarpkaya and Isaacson (1981). For non-linear effects to be included, a perturbation method has been used by Stokes expanding the first-order solution and involving higher-order terms to account for non-linearity. Despite this method allowing the inclusion of non-linear terms these weakly non-linear wave theories are also characterised by a fixed wave period and wavelength. Benitz *et al.* (2015) summarised in their review the fundamentals of fluid flow including the derivation for linear and weakly non-linear wave theories along with appropriate corresponding boundary conditions. In their work, different non-linear theories including Stokes, cnoidal and solitary waves are discussed.

In reality, waves are not regular, the wave height and period are not fixed, and thus the surface elevation will not be repeated as indicated by linear wave theory. This leads to the description of the sea state by means of a probabilistic approach which allows the representation of random, and more realistic, sea states with the use of a wave energy spectrum (Gōda, 2010). The two most common spectra, which are included in offshore wind energy standard codes BSI (2009) and DNV (2014), are the Joint North Sea Wave Project (JONSWAP) (Hasselmann *et al.*, 1973) and the Pierson–Moskowitz (PM). The latter was derived from measurements on weather ships and assumed a fully developed sea over a long period (Pierson and Moskowitz, 1964). The JONSWAP spectrum was developed from measurements in the North Sea and it introduced a peak enhancement factor. If the peak factor is equal to one, then the JONSWAP formulation is reduced to the PM spectrum.

2.2 Wave forcing

A popular engineering tool which is widely used for the determination of wave loads on offshore structures in the case of a slender body is the Morison *et al.* (1950) equation. For Morison *et al.*'s (1950) approach to be used, the structure must be slender. For the sake of clarity, a structure is considered slender if the ratio of the diameter of the cylinder to the wavelength, often called the diffraction parameter, is small ($D/\lambda \leq 0{\cdot}2$ where D is the diameter of the structure and λ is the wavelength). A slender body is usually referred to as a hydrodynamic transparent body which is often an assumption made for monopile OWTs. The total inline force $\boldsymbol{F}$ is given by the sum of the inertia and drag force. Furthermore, diffraction effects are ignored for a slender structure. An indication of the effect of viscous or potential flow effects is given in Figure 2 (Faltinsen, 1990). It should be noted that Figure 2 has limited applicability as it was derived from regular waves on vertical cylinders applying the Morison *et al.* (1950) equation (Equation 1) with $C_{\mathrm{M}} = 2$ and $C_{\mathrm{D}} = 1$. For a vertical slender cylinder, the wave force reads

1. $$\boldsymbol{F} = F_{\mathrm{D}} + F_{\mathrm{M}} = \frac{1}{2}\rho C_{\mathrm{D}} D u(t)|u(t)| + \frac{\pi}{4}\rho C_{\mathrm{M}} D^2 \dot{u}(t)$$

where F_{D}, F_{M} are the drag and inertia-induced forces, respectively; D is the diameter of the pile; ρ is the water density; C_{D}, C_{M} are the drag and inertia coefficients and $u(t)$, $\dot{u}(t)$ are the water particle velocity and acceleration, respectively.

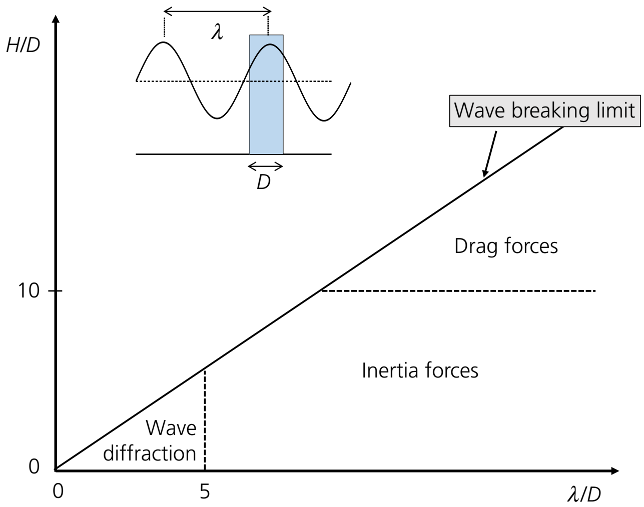


Figure 2. Wave-induced loading on structures (after Faltinsen, 1990)

It should be emphasised that selecting an appropriate wave theory is of great importance to determine the wave loads as wave kinematics contribute a significant part. For instance, when the linear wave theory is used, accurate estimation of the wave kinematics can only be provided up to mean sea level (MSL) due to the assumptions involved in its derivation. Correction techniques are often applied above the MSL. The so-called stretching methods, discussed by Chakrabarti (2005), are used in order to correct this uncertainty. Nowadays, the most widely used method is that proposed by Wheeler (1970), which, in practice, stretches the velocity profile up to the real surface elevation.

2.3 Wave breaking and run-up

Wave breaking is an event that has attracted significant research interest in the past years (e.g. Arntsen *et al.*, 2011; Wienke and Oumeraci, 2005). Breaking of waves occurs when the steepness of the incident wave has increased such that it leads to an unstable condition of the wave (Ochi and Tsai, 1983). Wave breaking can be a source of very high hydrodynamic loads; thus it is of high importance with respect to the loading on a structure. Phenomena such as breaking appear in both shallow and deep water and cannot be captured by linear wave theory and the semi-empirical Morison *et al.* (1950) formula due to the assumptions introduced (Chella *et al.*, 2012). Higher loads act on the monopile due to the violent impact on the structure. It can be considerably larger when breaking occurs at the moment of impact or just before the structure (Paulsen, 2013). Breakers are categorised into three types – that is, surging, plunging and spilling (Chella *et al.*, 2012). Of particular interest for monopiles are plunging breakers due to the very high loads they can induce on a structure (Wienke and Oumeraci, 2005).

The impact models developed by von Karman (1929) and Wagner (1932) are two pioneering models used to determine impact loads on piles. The difference between these models is attributed to the pile-up effect, which takes into account the flow around the pile. The pile-up effect is disregarded in von Karman's (1929) work which was implemented by Gōda *et al.* (1966) in their formulation for breaking impact forces. However, this effect was included in Wagner (1932) and according to Wienke and Oumeraci (2005) leads to a more accurate prediction of impact forces. Wienke and Oumeraci (2005) carried out experimental work on a large wave flume focusing on plunging breakers acting on a single cylinder for different inclinations and proposed an analytical model. The experimental and theoretical approach showed very good agreement and the importance of the pile-up effect was justified. In addition, it was indicated that the magnitude of the slamming force is strongly related to the distance between the location of the wave breaking and the structure. However, as this study focused on plunging breakers, the proposed model cannot be used for any type of breaker or varying bed slope. The analytical model superimposed the slamming force into the Morison *et al.*'s (1950) equation which is a current recommendation by the BSI (2009) offshore standard.

Another phenomenon which can be accounted for the structural damage on monopiles is wave run-up, which refers to the maximum wave elevation on the structure. It is of particular interest when it comes to platforms of monopiles. Implications were reported in the Horns Reef wind farm where the platform was damaged due to wave run-up (e.g. Bredmose and Jacobsen, 2011; Lykke Andersen *et al.*, 2011). One of the first attempts to determine run-up on structures of different geometry is noted in the experimental work driven by Hallermeier (1976), who proposed that the maximum wave height on the structure can be calculated from the velocity head in the Bernoulli equation. Kriebel (1990) extended the linear diffraction theory to second order to investigate the free surface elevation around vertical cylinders. It was found that the linear approximation can underestimate wave run-up up to 50% compared with that of the second order. Further work by Kriebel (1992) was carried out on the validity of second-order theory for wave run-up. It was claimed that the non-linearity included in the second-order theory is sufficiently representing the wave run-up in most of the cases. On the contrary, linear wave diffraction is significantly underpredicting the maximum wave elevation. Niedzwecki and Duggal (1992) performed a series of experiments to investigate the wave run-up on cylinders subjected to both regular and JONSWAP-based irregular waves. The results obtained from this study are in agreement with the findings of Kriebel (1992) that linear diffraction can be a good approximation for wave forces, but underestimates wave elevations unless the wave steepness is low.

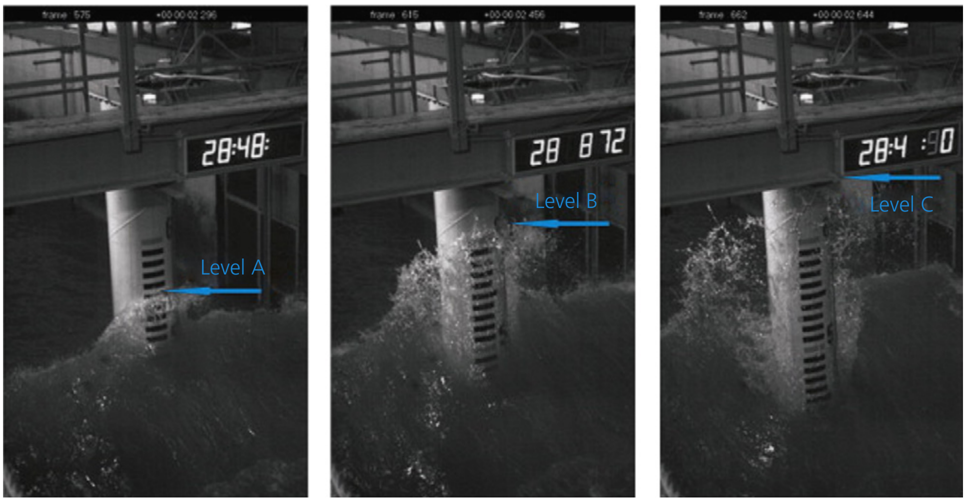


Figure 3. Wave run-up levels. Level A, the water is attached to the pile, Level B, mixture of air and water layer no longer attached to the pile and Level C, the level of maximum spraying (from Ramirez *et al*., 2013). (Reprinted from Coastal Engineering, **72**, Author(s), Large-scale model test investigation on wave run-up in irregular waves at slender piles, 69–79, Copyright (2013), with permission from Elsevier)

Later on experimental work focusing on the wave distribution and wave run-up around a monopile and a gravity-based foundation has been conducted by De Vos *et al.* (2007) and Lykke Andersen *et al.* (2011) leading to analytical equations for run-up. The accuracy of the measurements for wave run-up was acknowledged to require further improvement as it was noted that surface gauges cannot capture this phenomenon accurately. To address that and the potential scale effects, Ramirez *et al.* (2013) carried out large-scale tests to investigate the wave run-up and induced loads on platforms. It was claimed that the analytical formulae proposed by Lykke Andersen *et al.* (2011) ought to be adjusted. On the basis of the wave type improved factors for different run-up levels as shown in Figure 3 should be provided.

An evaluation of the existing methods was published recently by Kazeminezhad and Etemad-Shahidi (2015) suggesting a new method based on the analysis of experimental studies that predict level A run-up. The equations were derived by analysing available datasets with regression techniques and do not require precalculation of the wave kinematics which is a significant advantage when compared with previous analytical approaches.

2.4 Wave-induced loading and simulation tools

Currently, numerous aero-hydro-servo-elastic simulation codes (e.g. FAST, BLADED, Horizontal Axis Wind turbine simulation Code 2nd generation (HAWC2) etc.) that allow determination of the dynamic response of a monopile due to waves are used. In Robertson *et al.* (2015), a comparison between several codes is presented focusing on the hydrodynamics. Morison *et al.*'s (1950) equation was used to determine the forcing on a fixed cylinder under regular and irregular waves. The most notable finding in the scope of this research was the increasing non-linearity of the wave with increasing wave height and the importance of selecting higher-order wave models to predict the wave forces. It must be noted though, that in the study there was only a cylinder that was subjected to waves and no turbine was present.

A promising alternative in recent years to determine the loads on structures is computational fluid dynamics (CFD). One of the advantages of CFD is that the non-linear behaviour of waves can be captured more accurately than the popular simulation tools. Christensen *et al.* (2005) used a finite-volume CFD code using the interface capturing volume of fluid (VOF) method (Hirt and Nichols, 1981) to investigate the effect of two-dimensional regular waves on a monopile foundation. The effect of breaking waves and the wave run-up was presented; however, the viscous effects were neglected and only regular waves were investigated. In the results presented by Christensen *et al.* (2005), it was shown that the type of breaker is of significance, with plunging breaking waves leading to highest forces. Christensen *et al.* (2007) compared wave loads on a monopile at shallow water depths determined from undisturbed wave kinematics and the Morison *et al.*'s (1950) equation, and compared them with those acquired from CFD. Regular and irregular waves were examined. For regular waves it was shown that Morison *et al.*'s (1950) method provided satisfactory results for small wave heights, but for irregular waves it was shown that it is far more conservative, with the largest loads showing significant differences from the CFD results. The authors suggested that the loads from breaking waves cause significantly larger loads, but that was supported by only one test case. Bredmose and Jacobsen (2010, 2011) investigated wave impacts from breaking waves and the run-up using the VOF method. A particular focus of this paper was the vertical forces on inspection platforms from wave run-up. Focused wave groups were used in order to generate breaking waves; nevertheless, it is of importance that this method cannot generate severe events. Bredmose and Jacobsen (2010, 2011) confirmed the importance of these events and pointed out that Morison *et al.*'s (1950) equation cannot account for these loads, because the loads from undisturbed wave kinematics are determined as if the structure was absent.

Paulsen's (2013) research focused mainly on identifying how linear and non-linear potential flow solvers perform against CFD solvers in terms of accuracy. Special attention was paid to the determination of wave loads from steep irregular waves on a turbine's foundation located at a slopping bed. It must be noted though that the linear model did not perform well under the slope where the non-linearity of a wave is more pronounced. When it comes to the non-linear potential flow and the CFD solver, both predicted the inline force accurately; however, the CFD solver proved more suitable for high-steepness waves.

2.5 Current loading

In addition to previous loads, the water current which is often caused due to tides can result in an additional drag force on offshore structures. In terms of OWTs, the current loading is often considered as independent of time. According to the offshore wind standard DNV (2014), the current-induced forces can be treated with the Morison *et al.* (1950) equation using a mean or depth-varying current velocity. For further information regarding currents and current–wave interaction, see for example, Chakrabarti (2005) and Hogben and Standing (1975).

2.6 Aerodynamics

The need for advanced structural design has been enhanced by the global growth of the wind industry. A variety of models exist and have been used by researchers and industry for aerodynamic calculations. These models differ in terms of accuracy and computational cost from the simplest blade element momentum (BEM) approach to the most advanced CFD techniques. Modelling a monopile OWT requires calculations for different load cases involving environmental loads as described by the wind energy standard codes (BSI, 2009; DNV, 2014). Most of the engineering simulation tools used today to investigate the dynamic response of wind turbines use the BEM method (e.g. Failla and Arena, 2015; Hansen and Madsen, 2011).

BEM (Glauert, 1935) is a combination of momentum and blade element theory and it is the simplest and less expensive method in terms of computational cost (e.g. Failla and Arena, 2015; Hansen and Madsen, 2011; Hansen *et al.*, 2006; Snel, 2003). BEM has been favourably received for the calculation of aerodynamic loads and for studying rotor aerodynamics on horizontal axis wind turbines for many years. BEM is usually incorporated into aeroelastic codes, which are highly popular for modelling wind turbines. Although this approach can lead to satisfactory results for certain cases it is governed by numerous simplifications. Possible drawbacks arise from the fact that BEM models the rotor as a disc and assumes steady-state and axial two-dimensional flow (e.g. Hansen and Madsen, 2011; Hansen *et al.*, 2006; Rasmussen *et al.*, 2003). These assumptions introduced in BEM are often compensated by a number of sub-models (dynamic stall, dynamic inflow, tip loss, yaw and tilt) to allow for a more accurate aerodynamic modelling. Further information on BEM can be found in Hansen and Madsen (2011), Hansen *et al.* (2006), Rasmussen *et al.* (2003) and Snel (2003). One major disadvantage of the BEM method is that it requires tabulated data for C_l and C_d which are the drag and lift coefficients, to determine the lift and drag forces.

Vorpahl *et al.* (2014) compared the aerodynamic results obtained by different codes. The main differences between them arise from the fact that the loads in each code are applied to the deflected or the undeflected position of the blade. Moreover, various submodels used by the codes to address issues arising from unsteady aerodynamics lead to a differentiation in key loads (Vorpahl *et al.*, 2014).

More sophisticated and advanced numerical models for wind turbines based on the governing Navier–Stokes equations (NSe) were becoming increasingly popular in the literature as the computational cost has been reduced. Intermediate solutions between CFD and BEM are the actuator line (AL), actuator disc (AD) and the actuator surface (AS) models (Hansen and Madsen, 2011; Sanderse *et al.*, 2011). These models are coupled with the NSe and the blades are replaced by forces acting on the flow. Mikkelsen (2003) and the review paper by Sanderse *et al.* (2011) specifically focused on the AD, AL and AS models, which can reduce the computational cost significantly due to the simplified rotor used. The difference in these three models was on the distributed forces. The forces are distributed along a disc for the AD model, on a line along the blade or on a surface for the AL and AS models, respectively. The AD model has been used to study wind turbine wakes (e.g. Castellani and Vignaroli, 2013; Sørensen *et al.*, 1998). Another advantage of this model is that it can simulate many turbines at the same time (Miller *et al.*, 2013). The AL and AS models have been extended and more advanced and sophisticated models compared with the AD were presented by Sørensen and Shen (2002) and Shen *et al.* (2009).

3. GCs on monopiles

GCs are commonly used in monopile substructures and are extensively used in order to connect the monopile with the TP. The connection is formed by filling the annuli between the tubes with high-performance grout (Schaumann *et al.*, 2010). The purpose of the GC is to transfer the loads from the tower to the monopile. Figure 4 shows a schematic diagram of a GC. Unsatisfactory performance of the connection in several wind farms across Europe has been reported by several authors (e.g. Dallyn *et al.*, 2015; Klose *et al.*, 2012; Lotsberg *et al.*, 2012). Unexpected settlements of the TP and insufficient performance are issues that demand immediate attention. According to the offshore standard by DNV (2014), the attachment of the TP to the monopile can be achieved either by plain or GCs with shear keys. Shear keys refer to weld beads located along the circumference of the monopile and the TP. The main purpose of them is to provide additional resistance against sliding and thus enhance the connection's strength. That is because the use of shear keys along with their geometrical characteristics, such as spacing, height and material properties defines the connection's strength (DNV, 2014).

Along with geometrical considerations, several aspects of the connection regarding length or distance from the MSL are of importance and need to be considered during the design as they can affect the connection. A GC with and without shear keys is shown in Figure 5. In the following two sections experimental and numerical investigations on GCs are presented concisely.

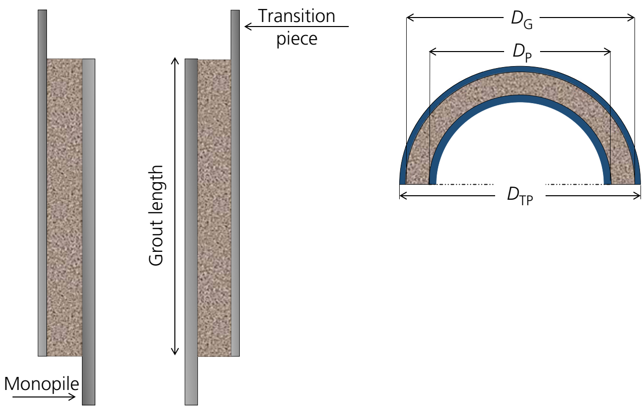


Figure 4. Characteristics of a tubular GC where D_G, D_P, D_{TP} are the diameters of the grout, monopile and TP, respectively

3.1 Experimental research

GCs were traditionally used in oil and gas structures (Figure 6) and have been of interest to many researchers since the 1970s (e.g. Andersen and Petersen, 2004; Billington and Lewis, 1978; Billington and Tebbett, 1980, 1982; Harwood *et al.*, 1996; Krahl and Karsan, 1985; Lamport *et al.*, 1991; Lotsberg, 2013; Schaumann *et al.*, 2010). A comprehensive review of the experimental work conducted to date was recently published by Dallyn *et al.* (2015). The purpose of the present paper is not to review this work; hence only some benchmark studies along with the most recent advances will be presented and subsequently the focus will be on numerical modelling. Figure 7 summarises the significant test campaigns that took place from the early 1970s until recently and how these affected the standard codes and design practice guidelines for GCs.

Aiming to establish a design procedure, the UK Department of Energy (DEn, 1980) initiated a research project to investigate GCs. Billington and Lewis (1978) and Billington and

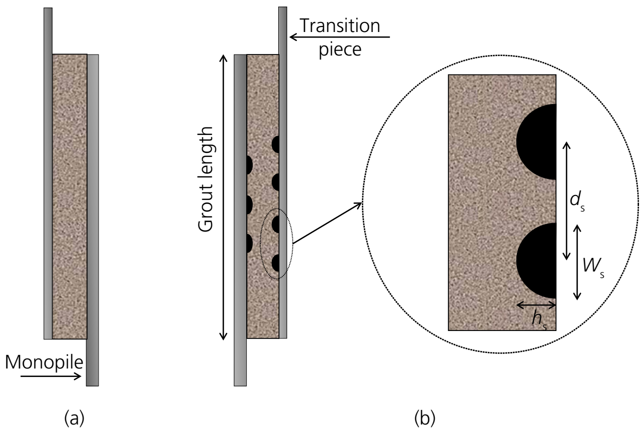


Figure 5. Tubular annuli grouted joint (a) without and (b) with shear keys where d_s is the distance from centre to centre of shear keys. W_s the width and h_s is the height of the shear key

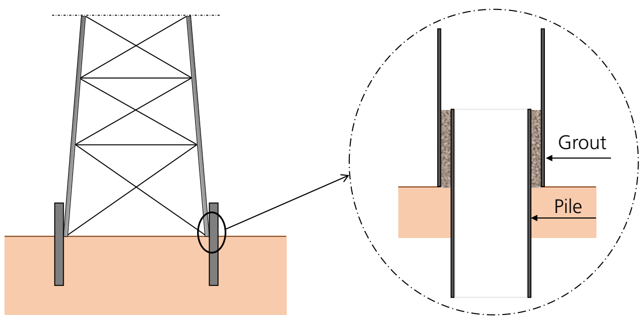
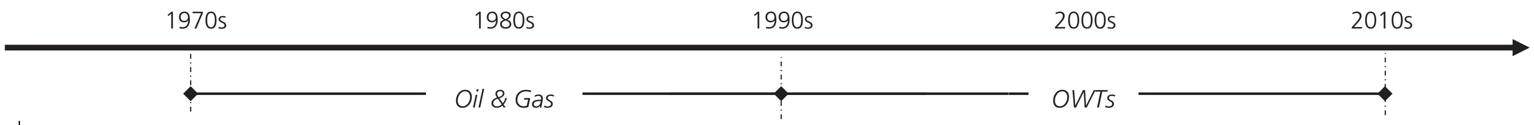


Figure 6. Offshore pile to sleeve connection



	1970s	1980s	1990s	2000s	2010s
Design basis	Design for GCs is based on [1]API	[4]DEn	[4]DEn, [10]API, [11]DNV		[24]ISO, [25]DNV, [15]API
Motivation	[1]Unsatisfactory design recommendations for GCs with shear keys [1]based on small-diameter GCs [1]Lack of guidance on the capacity of GCs with shear keys	Reconsideration of [1]API [5,6]New design process for API code on GCs [8]Development of DNV code for fixed offshore structures	[12]Assessment of the existing design equations Development of ISO standard code for offshore structures	2000: Design phase of Horns Rev I Wind Farm 2009: Slippage of TP in several OWTs Lack of large-scale testing and effect of bending moments remain unknown	[29–31]Effect of wet conditions is unclear [17–20]Bending moments are of significance and are introduced in the testing procedure
Action	[2,3]Experimental testing on axial capacity of GCs at Wimpey Laboratory, UK	[7]Analytical model for GCs with shear keys under axial loading [8]Axial capacity tests for DNV draft offshore standard code	[13]Review of 30 model test for ISO design equations [12]Experimental testing for a new analytical model	[16]Aalborg University model tests [17–20]Research project GROW, Hannover [17–20]Small-and large-scale testing of GCs [21,22]JIP I	[24]New recommendations for GCs with shear keys [27]Guidelines for GCs with shear keys [23]JIP II: fatigue testing, new design methodology for cylindrical shear-keyed GCs
Outcome and code development	[2,3]Significant amount of test data, new design equations on the strength of axially loaded GCs → Revised [4]DEn	Revised: [10]API, [11]DNV	[12]Combined loading does not have a detrimental effect on the axial capacity of GCs [15]API	Design equations do not represent actual capacity → Amendment of GCs section in [26]DNV [21]Design methodology for conical GCs	[27]DNV [28]Technical note on GCs [29–31]Large-scale cyclic tests under wet and dry conditions

[1]API (1977), [2]Billington and Lewis (1978), [3]Billington and Tebbett (1980), [4]DEn (1980), [5]Karsan and Krahl (1984), [6]Krahl and Karsan (1985), [7]Lamport *et al.* (1987), [8]Sele *et al.* (1989), [10]API (1984), [11]DNV (1989), [12]Lamport *et al.* (1991), [13]Harwood *et al.* (1996), [15]API (2000), [16]Andersen and Petersen (2004), [17]Schaumann and Wilke (2006), [18]Schaumann and Wilke (2007), [19]Schaumann *et al.* (2010), [20]Wilke (2014), [21]Lotsberg *et al.* (2012), [22]Lotsberg *et al.* (2013a), [23]Lotsberg *et al.* (2013b), [24]BSI (2007), [25]DNV (2010), [26]DNV (2007), [27]DNV (2014), [28]Lloyd (2013), [29]Schaumann *et al.* (2014a), [30]Schaumann *et al.* (2014b), [31]Schaumann and Raba (2015)

Figure 7. Evolution of GCs and standard codes

Tebbett (1980) conducted 400 tests focusing on the investigation of the effect of several parameters, such as grout properties and grout length to diameter ratio, on the connection's strength and derived the design equations. Important findings with regard to the bond's strength were drawn. Bond strength is the ultimate axial capacity over the surface area of the connection's interface. Billington and Lewis (1978) and Billington and Tebbett (1980) stated that the bond strength is proportional to the compressive strength of the grout. Furthermore, based on their findings they proposed the reduction of the bond's strength safety factor. Billington and Tebbett (1982) continued with further small-scale investigations on the fatigue life of the pile to sleeve connections and they acknowledged scale effects. However, it should be highlighted that while the experimental work is fundamental for GCs, the amount of data generated for fatigue life is limited and the compared plain and shear key connections were of different lengths due to set-up limitations. Fatigue behaviour of connections with shear keys was also examined by Boswell and D'Mello (1986). Nevertheless, only one geometry was investigated in the frame of this study and the emphasis was given on the grout's strength and therefore the results presented are of limited significance.

Similar to the work of DEn (1980) for plain shear key connections, Karsan and Krahl (1984) and Krahl and Karsan (1985) undertook a research project that led to design equations. These equations were developed according to the American Petroleum Institute (API, 1984) guidelines. Their data analysis justifies the conclusion by Billington and Tebbett (1982) that the safety factors used at that time were not correctly considered and the proposed relationship between compressive strength and bond strength for the grout introduced by Billington and Lewis (1978) was questioned.

Elnashai and Aritenang (1991) presented a non-linear numerical model which showed good agreement with experimental results and DEn (1984) equations, but the comparison with API (1986) deviated significantly. In another study by Lamport *et al.* (1991), special attention was given to the safety factors on the API (1986) and DEn (1984) which were also the subject of discussion in previous studies already discussed. The authors also confirmed the findings from Aritenang *et al.* (1992) that DEn (1984) formulation showed the best agreement with their experimental data. This was also noted in the review by Dallyn *et al.* (2015) along with the realistic manufacturing procedure carried out by Aritenang *et al.* (1992) for the specimen used in their experimental tests. Later on Harwood *et al.* (1996) conducted experimental work for International Organisation for Standardisation (ISO) guidelines including proposed values for the height to distance (h_s/d_s) ratio for shear keys. However, it has to be noted that this work was also conducted in a small-scale set-up, which was previously acknowledged by Billington and Tebbett (1982) to have an effect on the connection.

Andersen and Petersen (2004) presented large-scale experiments using specimens of 1:8 scale. The specimens were designed to be similar to connections used in the Horns Rev wind farm and were subjected to bending. This research work was conducted at the University of Aalborg using high-strength grout and focused on extreme and fatigue loads. On the basis of the data documented, the specimens were found to have sufficient performance with no cracking occurring and only a small gap forming between the grout and the steel for the ultimate load. A numerical model was also presented. Yet in the research work by Andersen and Petersen (2004), the environmental loads applied both numerically and experimentally were not discussed in detail and this was also highlighted by Dallyn *et al.* (2015), where the loading regime was questioned. It is worth mentioning that the shear keys were only installed on the upper and the lower part of the overlap length of the connection.

Schaumann and Wilke (2006) conducted small- and large-scale experiments for GCs using high-performance concrete and their investigations concluded that shear keys should be used for GCs, as they enhance the performance of the connection significantly. The urgent need to address these issues initiated two joint industry projects by DNV (JIP I, JIP II (Lotsberg *et al.* 2012)). The first research project focused on the axial capacity of plain GCs and the main findings are presented in Lotsberg *et al.* (2013a). It was found that plain connections without shear keys were no longer recommended for monopiles as the effect of the large diameter on the capacity is more pronounced than the one accounted for in the design. The finding of insufficient performance of plain cylindrical connections is in agreement with the findings of Schaumann *et al.* (2010). A new conical shaped connection was proposed by Lotsberg *et al.* (2012) in order to limit the settlements of the TP. A second joint industry project which took place from 2011 to 2012 focused on cylindrical shaped connections with shear keys (Lotsberg *et al.*, 2013b). The project aimed at developing a design methodology for shear-keyed connections. In an attempt to address scale effects, the test specimen should be designed in order to have realistic structural properties as in the monopiles installed offshore. Lotsberg (2013) described the derived analytical procedure developed for the ultimate limit state (ULS) and fatigue limit state (FLS) along with the design of the specimen for realistic structural properties.

Wilke (2014) carried out large- (~1:6·25) and small-scale tests on GCs for monopiles. Plain and shear-keyed specimens were investigated under bending. Along with Andersen and Petersen (2004) these are the only experimental campaigns undertaken to investigate GCs under bending. Wilke (2014) suggested that the application of shear keys is necessary as their application reduces damage in the circumference of the grout and the damage is minimised for the connection in ULS and FLS.

3.2 Numerical modelling

Numerical modelling of GCs can provide a promising alternative to the experimental testing since physical tests are very expensive, but several aspects of the model need to be considered carefully in order to achieve reliable results. Andersen and Petersen (2004) summarised the main challenges arising in such models, mainly those occurring due to the non-linear behaviour of the grout, the steel–grout interface and due to the element type used. The effect of the steel–concrete interface and the highly brittle behaviour of the grout are highlighted by Nielsen (2007), who addressed practical modelling issues of finite-element (FE) models of GCs. In terms of FE models of monopiles, these usually consist of three parts – that is, the TP, the grout and the monopile. It has to be noted that when it comes to full-scale models the accuracy and the computational cost of the model ought to be considered carefully.

The work carried out by Andersen and Petersen (2004) was further extended with FE analysis of the experimentally tested models. In their numerical work, the authors applied the Drucker–Prager model to account for the non-linearity of the grout material. Taking into account the non-linear behaviour of the grout is of high importance and particularly under compression (Lohning *et al.*, 2013). Overall good agreement was found between the numerical and experimental data. The forces were shown to transfer as a force couple in the grout's top and bottom regions.

In contrast to the sufficient performance of plain GCs documented by Andersen and Petersen (2004), Schaumann and Wilke (2006, 2007) recommended that shear keys should be used in all GCs due to the enhanced strength they provide. In the numerical work that was carried out special attention was given to material models with respect to local (e.g. local shear key analysis) or global (whole GC) interest on the connection. Furthermore, in order to avoid penalising design decisions, Wilke (2014) recommended the application of mechanical shear keys in the centre region of the connection and not in the whole length.

Gjersoe *et al.* (2011) investigated numerically the effect of interface behaviour between grout and steel and its contribution to potential grout cracking and proposed the use of packers to restrict the grout and increase confinement. The numerical work presented though focused on plain GCs, however, only linear properties have been used for the grout. With regard to the loading regime, the applied loads were taken from a full-scale 3·6 MW turbine. Prakhya *et al.* (2012) reviewed the current practice guidelines and the design equations provided by DNV (2007) before the joint industry projects and carried out numerical work based on a simple proposed model for moment transfer.

In Löhning *et al.* (2013), an FE analysis of a GC was carried out in order to identify the mechanism of the settlement of the TP. The available models to simulate the connection's interface were addressed. Moreover, a plain cylindrical GC is found to be insufficient anymore for monopile OWTs. In addition, the findings from this study suggest that the use of shear keys is recommended and this is in agreement with the findings of other researchers (e.g. Schaumann and Wilke, 2006; Wilke, 2014).

4. Discussion

This paper presented the state of the art regarding GCs on monopile OWTs aiming to identify the uncertainties in the design of the connection and the environmental parameters affecting it. The determination of the hydrodynamic loads acting on a monopile with the wave and current theories used today as well as those used to determine the aerodynamic loads on an OWT were discussed. Computational techniques involving aero-hydro-servo-elastic simulation codes along with CFD tools were addressed in terms of their capabilities and efficiency to determine the structural response of a monopile.

As demonstrated by the reviewed papers, CFD tools can be used to model highly non-linear events such as breaking and produce high-fidelity results; however, the computational cost is significantly increased. On the contrary, OWT simulation codes using the linear wave theory along with the Morison *et al.* (1950) equation can provide adequate results without requiring significant computational resources. Nevertheless, it should be noted that severe non-linear events cannot be captured.

From the reviewed publications it is evident that wind loads are often effectively treated by means of the BEM method. Incorporating the appropriate submodels enhances significantly the accuracy of the method. Additionally, BEM-based codes are computationally efficient and easy to implement in the design of OWTs.

With regard to the experimental and numerical work on GCs, the current advances on the design of the connection have been thoroughly discussed. Furthermore, the uncertainties arising when simplifications are introduced in numerical models have been addressed. The lack of test data from large-scale tests on GCs supports the notion that the design of the connection has been based on the experience collected from the oil and gas sector. More specifically in several research efforts, the loading conditions did not resemble those experienced in reality by monopiles. This is particularly pronounced by the constant evolution of the developed standard codes and guidelines during the past decade.

FE analysis may contribute significantly towards understanding the structural behaviour of GCs. It is clear, however, that the unique characteristics of the connection introduce a number of implications. More accurate concrete models ought to be developed for the grout in order for the actual behaviour of the material to be captured. One of the challenges for future research will be to represent adequately the behaviour of the grout.

5. Concluding remarks

From the reviewed literature it is reasonable to conclude as follows.

- The linear wave theory along with the widely used Morison *et al*. (1950) equation can be sufficiently used for environmental conditions where normal sea states are of interest.
- The OWT simulation codes, involving linear, weakly non-linear wave theories and BEM can be of great assistance regarding OWT modelling. Their ability to model the dynamic response of OWTs subjected to wind, waves and current without penalising computational cost is a significant advantage when compared with CFD tools.
- Despite the recent progress on the understanding of GCs there is still lack of test data derived from experimental campaigns with representative offshore environmental conditions and monopile configurations.
- In terms of FE analysis of GCs, the highly brittle, non-linear grout behaviour and the interface behaviour between the grout and steel ought to be carefully included in the numerical models in order to represent realistically the load-transfer mechanisms between steel and grout.

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